NEW YORK STATE
DEPARTMENT OF TRANSPORTATION

GEOTECHNICAL UPDATE REPORT
SR 443 Delaware Avenue Landslide
Elsmere, New York

JUNE 26, 2000

LANDSLIDE TECHNOLOGY

REPORT

GEOTECHNICAL UPDATE REPORT
SR 443 LANDSLIDE
ELSMERE, NEW YORK

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GEOTECHNICAL UPDATE REPORT
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1. BACKGROUND

1.1 General

1. The accounts of the first week of slide movement and subsequent initial geotechnical investigations are described in our Concepts Report, which was submitted to the New York State Department of Transportation (NYSDOT) on June 5, 2000.
2. The drill crew was in the central area of the slide (Boring 5) in late May. By Wednesday May 31st, after the samples were obtained, the ground to the east was observed to be moving more actively and therefore the drill crew abandoned the site and delayed installing the inclinometer casing until the ground movements became less threatening.
3. It was decided to unload material from the top of the headscarp to try to slow the slide movements to make it safer for the drilling operations below. By June 5th, the NYSDOT had trimmed back the steep slope. The excavation was intended to produce a slope of 2H:1V in the upper unit of brown varved clay.
4. On Tuesday, June 6th, a significant rainstorm occurred, resulting in a record 3 to 5 inches of rain in one day. The cut slope experienced localized shallow failures and sloughing. To stabilize the cut slope, rock fill was delivered and pushed down the slope to protect against further erosion. A thin blanket of rock was intended.
5. At approximately 1pm, Saturday June 10th, the landslide toe slumped / flowed into the relocated stream and enlarged to the east. Repair work in the headscarp area was halted. The entire landslide moved, which impacted several dozers and a drill rig. The City of Albany subsequently excavated the debris from the relocated stream channel and added more riprap to the north bank of the channel.

6. On June 12th, the width of the stream channel was measured by NYSDOT, using a range finder.

a.	At waterline crossing opposite concrete HW (upstream end)	115 ft
b.	At 1st channel bend by clump of trees	57 ft
c.	At 2nd bend in stream by start of additional riprap stone	72 ft
d.	At end of additional riprap stone lining	96 ft
e.	At 3rd bend, 165 feet west of bank tree line	98 ft
f.	At end of garden fence, 45 feet west of bank tree line	116 ft
g.	At end of garden fence, (to O.G. south bank)	150 ft
h.	At power line / gas line (downstream end)	95 ft

7. The New England news media reported that a small magnitude earthquake (M3.5?) occurred during the night of June 15/16 in western Massachusetts / Connecticut. However, there was no mention in the Albany newspapers of any ground motions in the Bethlehem area.

1.2 Additional References

1. Additional references were provided by NYSDOT that included 1) examples of historic landslides in the vicinity, 2) soil strength test data on varved clay from the local area, and 3) information on water-bearing materials within or beneath the clays. The references include:
   a. Moran, Proctor, Mueser & Rutledge Consulting Engineers, "Review of Foundation Conditions for the South Mall Project, Albany, NY," December 1963.
   b. Regan, P., "Landsliding In The Hudson Valley With Particular Emphasis On The Mahar Slide (Bethlehem, NY) Of June 1968," New York State Geological Survey, January 1982.
   c. Fickies, R., "Landslide – Bethlehem near Normans Kill Creek," University of the State of New York, June 28, 1982.
   d. Hoffman, V., "Geotechnical Report For Earth Slide, Delaware Avenue, Delmar, NY," October 28, 1982.
   e. Mueser Rutledge Consulting Engineers, "Dock Investigation and Rehabilitation Study, Port of Albany, NY," April 30, 1985.

2. The Mahar slide in Bethlehem (west of New Scotland Road) occurred in 1968. "The resulting scarp, which reached a maximum height of 70 feet, is 1000 feet long and is inclined 60 degrees." "A portion of the Normans Kill was displaced 300 feet to the north. A slide blocked the creek and caused flooding of nearby low lying areas." The slide plane was inferred to be horizontal at an elevation of +85 feet. The groundwater level ranged from elevation +170 feet and +189 feet near the headscarp. The recent slides have been considered a reactivation in ancient slide terrain, where new scarps line up closely with historic slump scars. "Landsliding in Lake Albany clays was initiated long ago and has been sustained by fluvial undercutting of slopes."

3. A landslide occurred in 1982 in Elsmere, between Delaware Avenue and the Normans Kill creek, to the west of the new slide. The slide was 320 feet wide and about 400 to 500 feet long. An old shear zone of varved clay, underlain by a thin seam of coarse silt, was interpreted from exploration samples at an elevation of about +85 feet.

2. JUNE 15TH SITE RECONNAISSANCE

1. We performed a geotechnical reconnaissance of the enlarged slide area on June 15, 2000. The landslide headscarp, toe, and sidescarps were traversed. Also, the slope to the east of the slide was walked to look for slide features. The main landslide had enlarged to the east about 20 to 40 feet, where new slumps were evident. This area had been identified in our previous reconnaissance as being marginally stable, with tension cracks extending east from the main slide sidescarp and several slump blocks in a partial state of collapse. Figure 1 presents photos showing some of the new slide features.

2. We observed new survey stake lines running from the top of the slope to the stream, north of the Hoffman's Jiffy Lube building. A sag pond still exists just below the Jiffy Lube gabion wall.

3. In the power line easement, several cracks and zones of seepage were identified. The upper tension crack was located about a third of the way down from the top of slope (about 30 feet down), downslope about 10 feet from a significant zone of seepage. The seepage area was more pronounced following drill access road activity, which caused the surface to pump and liquefy.

4. One of the dozers was stuck (halfway submerged) in apparently liquefied mud along the slumped portion of the east sidescarp. Another dozer was observed to be

straddling a tension crack in the slumped area about 30 feet downslope of the toe of the headscarp.

5. The recent slide movement exposed the lower portion of the headscarp, about 20 feet vertical. Ponded water was observed in the depressions formed by the recent slide activity. Rockfill and mud were evident on the upper portion of the slumped slide mass. The mud apparently came from the eroded scarp slope, as well as from being pushed down by placement of the new rockfill. The rock material had been used to stabilize the trimmed headscarp slope, but slid out when the main slide mass moved June 10th. Some of the material appeared to have been dozed prior to June 10th onto the middle portion of the slide mass, possibly to regrade the slide area so it could drain surface water away. The rockfill had been dumped from the highway curb area and pushed down the headscarp (south and southwest locations). The bottom of the rockfill ramp extended onto the upper slumped slide mass, in the southwest corner of the landslide. The highway culvert on the eastside was fitted with a flexible plastic pipe: this flexible pipe had come apart at a joint after the main slide moved June 10th. The culvert was plugged in late May, therefore no water flow was evident in the culvert or drainpipe.

6. The central slide mass appeared to have translated horizontally about 50 feet or more. This estimate is based on the new positions of the drill rig and the upper tilted blocks of gray clay.

7. The lower slide mass appeared to be higher in elevation than the native ground to the west side. The disturbed slide debris was forming a higher ridgeline along the lower slide boundary.

8. The City of Albany's contractor was removing slide debris from the stream channel and was placing additional riprap. The stream channel alignment was irregular because of slumped and eroded materials from the toe of the slide, and from excavations for new riprap, which extended further north midway along the slide-impacted section of stream bank. Photos of the stream channel conditions are shown on Figure 1.

3. FIELD AND LABORATORY DATA

3.1 Survey Monitoring

1. Many of the survey targets in the active landslide have been damaged. Therefore it was not possible to measure actual displacements of the landslide.

2. On the east side of the slide, survey targets S1 and S2 show that the ground moved about 0.2 to 0.7 feet in the 1 to 2 days encompassing the June 10th slide movement. The ground movements then leveled off for a few days, but showed slightly increased movement rates on June 15th. Points S3 and S4 moved about 5 to 6 feet on June 10th, but have not shown movement through June 13th (these points might have become damaged and therefore have not being measured since June 13th).

3. Survey Points H1 and H4, near the north side of the Jiffy Lube building, indicate less than 0.15 feet of apparent movement between June 7th and 9th and visibly 'no movement' from June 9th to June 15th. Survey target H2, slightly downslope of the gabion wall, is showing a greater movement trend of about 0.7 feet (approx. 0.1 feet per day).

3.2 Inclinometer Data

1. The inclinometer in Boring FHX-1, above the headscarp on the southwest, is not showing any movement (measured from May 22nd to June 13th). Also, the inclinometers installed in the borings above the central and eastern headscarp areas are showing no movement (FHX-7, FHX -8, and FHX -9, from June 5th to June 13th).

2. Other inclinometer installations were planned for the active landslide area, but have not been installed yet.

3. The pavement above the May 18th headscarp does not show any signs of slide-movement cracks or scarp retrogression.

3.3 Groundwater Data

1. The groundwater monitoring included measurements in several observation wells and piezometers, as well as observations of groundwater seepage.

2. Observation wells P2B and piezometer P7A, installed in the pavement near the road, measured groundwater about 20 to 26 feet below the road level (elev. +174 to +180 feet). The observation wells might not be sufficiently sensitive to measure short-term fluctuations in groundwater pressure, given the relatively low permeability of the Albany Clays.

3. The seepage evident in the headscarp and the slope where the powerline and gas pipeline easement crosses was observed to be about 20 feet below road level and lower.

4. Seepage in the slopes east and west of the slide toe mass was observed from the stream bank up to about elevation +130 feet (visual estimate).

5. Artesian groundwater was reported during drilling north of the relocated stream channel, at preliminary boring FHX-10. This boring was located on the north stream bank towards the east side of the slide limits, near the fenced garden plots. The artesian water level that was measured during drilling was about 10 feet above the ground surface, visually estimated by using drill pipe. It is not known at what depth the high water pressure was located.

6. Groundwater measurement in piezometer P6 to the north of the stream on June 12th was at approximately elevation +99 feet (top of stream bank).

7. There are currently no reliable groundwater level measurements of water head along the slide plane. Excess pore pressures (or artesian conditions) are suspected. Several piezometer installations are planned.

8. Most of the reference publications refer to sand layers under and within the Albany varved clays, where groundwater pressures have been reported to be significant. The 1837 landslide in Troy was caused by 'tremendous hydrostatic pressure' and flowed 800 feet like an avalanche. The 1916 slide at the Knickerbocker Portland Cement Company, outside Hudson, occurred after a heavy rainfall preceded for several days. The scarp was defined by a nearly vertical bank. "This uplift (at the toe) was accompanied by tremors and subsidence of the higher land to the west." ".... the lower fluid was able to flow out onto the flats east of the abandoned channel." "It appears the creek channel constituted a zone of weakness and was one of the essential causes of the slide. The most important element of the slide was the saturated substratum of blue clay." "The water appears to have come from lateral infiltration along the beds rather than from vertical seepage from the surface."

3.4 Laboratory Data

1. Laboratory testing has been in progress, primarily on samples from the borings located above the headscarp. Some tests have been performed on Boring THX-4 (central slide area). Most of the strength data are peak shear tests. Refer to the June 5th Concepts Report for a summary of test results. Recent test results have been incorporated into the summary table (at the end of this report). Preliminary review of the data confirms earlier test results. Further lab testing is in progress.

2. In general, the recent lab-tested peak effective stress friction angle is about 15 to 29 degrees, plus a cohesive intercept of 200 to 600 psf. The residual effective friction angle is interpreted to be about 18 degrees friction, based on the most frequent test results. The undrained (total stress) shear strength, Su, is typically at least 2000 psf to as much as 6000 psf peak (in overconsolidated clay). The remolded Su tests have not been done yet, other than a few unconfined remolded lab vane tests that resulted in low-end values of 200 to 300 psf. Correlations with overburden stress in the cited references suggest that the Su values for normally consolidated Albany Clays should be about 300 psf to 1000 psf. This 300 psf to 1000 psf range probably correlates between the lightly loaded toe area of the slide to the more heavily loaded upper slide blocks.
3. Direct shear tests performed for the 1982 slide to the west of the existing landslide showed residual shear friction angles of 25 to 30 degrees, on soils that were low to moderate in plasticity, i.e. 5% to 25% PI. These tests results appear too high when compared to the back-calculated strength for the new slide (discussed below).
4. Based on an assumed Factor of Safety (FS) of 0.95 for the June 10th slide conditions (and interpreted geotechnical parameters), the back-calculated average residual shear strengths for the basal failure plane are:
    a. Residual Su of approximately 500 psf (total stress analysis), or
    b. Residual effective friction angle of 18 degrees (effective stress analysis).

4. OPINIONS ON RECENT SLIDE ACTIVITY

4.1 General

1. The accounts of high groundwater or artesian pressures are plausible and should be accounted for in landslide stabilization analysis and design.
2. The slope between the highway and the Normans Kill to the east of the main landslide is marginally stable and should be closely monitored for signs of slide enlargement and impact to facilities. The power line and gas utilities should monitor this condition carefully and to prepare for protective measures and repair options.

3. The recent June 10th slide created an opportunity for improved back-analysis. Groundwater and shear strength assumptions could be further evaluated, assuming a range of possible slide plane depths.
4. Unfortunately, there were no piezometers or inclinometers installed yet in the active slide mass during the June 10th landslide movements.
5. The groundwater pressure changes associated with storm events, such as the record storm on Tuesday June 13th, appear to lag several days given that the renewed slide activity occurred about 4 days after the storm (this groundwater behavior is interpreted, since responsive groundwater level data is not available).
6. The observation wells (standpipe monitoring wells) are probably not sensitive to rapid groundwater fluctuations and could miss peak water levels. Vibrating wire piezometers would likely be better suited to detect water pressure changes.
7. The survey targets are a simple and successful means of measuring relative ground displacements and to verify which areas of the landslide are moving and to what degree.
8. The headscarp appears to be not retrogressing towards the highway. This indicates a lower risk of hazards if the highway were to be partially reopened.
9. The Normans Kill stream water can produce flows large enough to cause slope bank erosion problems, exacerbating landslide instability.
10. The conditions that affected instability during the June 10th slide event appear to be:
    a. Slope toe erosion from high stream water flows.
    b. A large storm event that resulted in great and sustained amounts of precipitation.
    c. Marginal initial slope stability.
    d. Soil shear strength that likely has become reduced to lower residual levels because of slide strains.
    e. Some added driving force due to the placement of new rockfill materials on the upper part of the slide mass.

4.2 Risks to be Addressed in Design Development

1. Erosion or piping, leading to slope failures, caused by seepage from the headscarp slope.
2. Excess pore pressures caused by movement & vibration of heavy equipment.

3. Significant rainfall (with inadequate drainage) causing higher groundwater pressures.
4. Stream erosion of slide toe, causing removal of slide toe resistance (at upstream end due to sharp bend in channel, and near downstream end where the relocated channel is adversely angled towards the original and existing channel).
5. Excavation of slide toe by the City of Albany contractor to restore stream channel width, causing removal of slide resistance.
6. Widening of landslide to the east due to marginally stable ground conditions, especially where groundwater pressures become elevated.

5. RECOMMENDATIONS

5.1 General

1. Install vibrating wire piezometers (with small confined porous tip zones) to accurately measure groundwater pressure changes on the slide plane. Install several piezometers near the toe and middle of the active slide mass in order to confidently develop a critical phreatic surface for design stability analyses and for monitoring slide performance during construction. It would be important to know what water pressures realistically exist that may be benefited by drainage, and how the water pressures behave so that drainage measures can be appropriately designed. The recommended location of new instrumentation is shown on Figure 2, along with recommended piezometer installation elevations. Since the slide plane elevation is uncertain (still being investigated), we recommend 2 to 3 piezometers per location to bracket the possible elevation range. Data loggers are recommended to continuously monitor (every 2 hours) changes in groundwater pressure, particularly when major storms occur. Normal manual monitoring might not be sufficient to capture the more significant peak pressures.
2. Research and clarify previous observations concerning groundwater conditions, particularly observations of excess water pressure or artesian groundwater. Document.
3. Install and monitor inclinometer casing within the lower area of the active slide mass. Inclinometer installations to the east of the main slide, at mid-slope, are also recommended, as shown on Figure 2. The areas include the Jiffy Lube property and the powerline easement, where both appear to be experiencing slide

movements. The slide plane geometry will affect the relative benefit of various stabilization options. If the slide plane geometry is assumed rather than measured, then the designs tend to become less effective and more costly.

4. Immediately install a tight grid of survey targets to measure changes in slide movement across the slide as well as beyond the current boundaries. This will allow good management of safety issues during drilling and construction activities in the slide area, as well as providing revised trends and directions of slide movement and growth.

5. Survey a new set of cross sections for stabilization design. The June 10th slide has significantly changed the ground surface and the relocated stream channel has been altered by new riprap efforts. Obtain surveyed cross-sections downslope of the Jiffy Lube and in the powerline easement.

6. Identify, stake, and survey the locations and elevations of groundwater seepage on and near the slide mass, the powerline easement, and downslope of the Jiffy Lube Gabion wall. The boundaries of seepage and wet areas to both sides of the slide toe should be staked and surveyed. The purpose would be to augment piezometer data and plot it on representative cross sections.

7. Decide whether to relocate the stream channel further to the north and to make the curves less abrupt (and therefore reduce erosive potential). This decision will impact how the lower half of the landslide is stabilized. We recommend shifting the stream channel to provide more positive toe stabilization and protection. Alternatively, additional resistance would need to be provided for the slide toe to compensate for loss of toe support and marginal slope stability resulting from excavation to widen the stream channel.

8. Perform laboratory undrained shear strength tests to simulate the residual failure plane. The results should be calculated to determine both Su (psf) and total friction strength angle ø (degrees). Perform triaxial shear strength tests on normally consolidated (remolded, then consolidated) samples of clay selected from the varves at the slide failure plane. Triaxial test specimens should be consolidated to the average isotropic overburden pressure (estimated about 8 and 15 psi). The amount of strain for each test should be large enough to confirm that the deviator stress has reached its lowest value (residual), well beyond the peak value.

9. Perform ring shear tests on remolded specimens of the clay selected from samples near the slide failure plan. Since NYSDOT does not perform this test, the Landslide Technology laboratory would be available to do so.

10. The observational approach for design and construction should be used since there will undoubtedly be several geotechnical uncertainties that will need to be evaluated during and following construction, to verify design and to allow for subsequent changes as needed for desired slope stability.

11. Excavate the remaining steep upper scarp slope, back to a line 5 to 10 feet north of the curb. Protect the exposed headscarp slope with an erosion control mat.

12. Provide riprap protection of the slide mass toe, to protect against short-term stream erosion. This may be a temporary measure if the final stream location is changed, but is necessary to protect the existing slide and surrounding property / facilities from greater impacts in the meantime. Also, if the stream channel alignment stays where it is currently, the riprap would need to be designed for long term stability and to resist scour.

13. Collect and drain all surface water ponding on the slide mass. Sumps pits and pumps will likely be needed. Discharge to the stream bank (tight-lined pipes).

5.2 Comments on NYSDOT Design Concepts:

1. Review of plan provided Thursday June 15th.
2. Concepts are rational and responsive to the new conditions observed.
3. Top priorities of immediate concern are to provide temporary protection to headscarp slope and to protect the slide toe from stream erosion.
4. For short-term stability, the headscarp slope should be unloaded an additional amount, to slope back to a 5 to 10-foot offset from the curb. About 200 to 400 cu. yds would be excavated and hauled away. Minimize construction vibrations (truck and backhoe) near the headscarp. The slope should be hydroseeded and then protected with an erosion control mat (woven). Willow wattles were considered initially, however, they likely would not provide benefit until next spring. Therefore, wattling was not considered further.
5. The slide toe should be protected with riprap as soon as possible. This is an emergency action to help control near-term erosion risks in the event additional storms occur prior to the final stabilization being constructed. The riprap would likely need to be placed by displacement methods, by building a rockfill access road around the perimeter of the slide toe. Construction should start on the west end to provide the most critical protection first. As the rockfill is pushed ahead, it will displace clays / muds in the stream channel and forms its own foundation. This procedure should be adequate for the short-term application of the

temporary riprap. Clay mounds or waves pushed into the stream channel would need to be removed to prevent obstruction of the channel. The hydraulics engineer should be consulted to determine the width of stream channel needed, the size / gradation of riprap rock fill, and foundation elevations to avoid scour.

6. If the stream channel must be made wider than the existing maintained minimum width of 35 feet, then the riprap should be constructed in a trenched approach. The excavation and backfill should be done in short segments to minimize the destabilizing effect of the excavation. We understand that the desired short-term stream channel width should be about 55 feet, based on an agreement between the City of Albany and NYSDOT. Therefore, the riprap section width should be 20 to 30 feet wide to make up for the lost support, caused by removing the slide toe mass to create the 55-foot channel width. The trench segments should be about 10 to 20 feet long and extend down to at least elevation +75 feet, then the trench segment should be backfilled with the riprap rockfill, and then the next segment would be excavated, and so on. The bottom lift of rockfill should preferably be a finer gradation to act as a filter blanket, or a geotextile could be used. Figure 3 presents a conceptual sketch. A concern during construction is that the excavations could encounter soft and water-bearing soils. The potential exists for 'bottom heave' and side-cut failures. We recommend that the trench segments be excavated 'under water', to counteract hydraulic uplift/seepage forces. The excavation segments should be made short so that back-filling can be made quickly. Trench excavations will also encounter softer slide debris, depending on the alignment of the riprap revetment. The longer time the soft/wet soil cuts remain open, the greater the risk for side cut failures. Filter material is recommended to prevent piping and subsidence. No workers should be allowed in the excavation trenches.

7. Since the temporary riprap might be eventually selected for long term use, the scour potential should be evaluated. The depth of riprap embedment should be made deep enough to resist significant storm events.

8. To reduce the potential for stream bank erosion, the north bank should be excavated near the east end of the slide to form a more gentle horizontal alignment. The east end of the existing ripraped north bank is curved sharply at a cluster of trees. The trees should be removed and the channel straightened.

9. Also, if the existing channel and temporary riprap are to be relied on long term (including next winter), the streambed should also be reinforced with a riprap

blanket. It is expected that the new 55-foot wide channel could experience high stream velocities, and therefore the revetment should be designed accordingly.

The length of the riprap should be designed to prevent secondary impacts, such as erosion of neighboring stream banks and behind the riprap, and to protect against new currents created by angles in the realigned channel.

Initial recommendations for the length of riprap are: a) to start upstream of the main channel bend, near the location where the 48-inch water conduit crosses the stream, and b) extend far enough downstream (east of the Powerline easement) to a point where the channel flow is parallel and linear to the stream bank.

The end points of the riprap should be reinforced. Consider the use of keyed riprap and possibly grouted riprap for improved riprap resistance. A trench backfilled with riprap rock should extend perpendicular to the stream bank at both ends of the riprap stream bank revetment, to prevent erosion behind the riprap revetment.

10. At the time of riprap construction, several French drains within the landslide toe mass could be initiated perpendicular to the stream bank and connecting into the riprap. These initial French drains would help to dewatering the outer toe of the slide and would be available as discharge points for subsequent subdrainage systems. The french drains should be wrapped in nonwoven, high strength geotextile and filled with coarse free-draining rock fill.

11. Immediately, determine relative ground stability in central and lower slide areas so that drilling and piezometer and inclinometer installations and slide depth determinations can be made quickly. Utilize survey targets to verify that ground movements are minimal and predictable. It is hoped that the drilling can proceed immediately and concurrent with riprap construction.

12. Provide for immediate drainage of surface water and ponds on the slide mass. Discharge the water in solid pipes to the stream's edge. Pumping will likely be needed, in areas where water is ponded.

13. The desire to reopen the highway to two lanes can probably be accommodated after the initial site work is accomplished, including headscarp excavation and protection, surface drainage, and initial riprap placement. Monitoring of the slide continuously is critical to knowing the level of hazards and if the hazards become significant to close down the lanes. This should be done with full-time safety personnel, continuous slide monitoring, and flaggers. Only passenger cars should be initially allowed, at slow speeds. Monitoring should include at least a row of

stakes along the north curb so that an inspector can detect if movements are beginning to occur simply by 'line of sight' displacement of any of the stakes. Also, a grid of survey targets should be monitored daily within the slide mass. If the slide becomes wider, as it appears to be doing to the east, then the risk of headscarp retrogression increases. Therefore continuous monitoring of the east slide boundary and adjacent slope is needed.

14. The concept of providing deep drainage, using sand-drain or wick-drain relief drains, is worth pursuing. However, more information is needed concerning water pressure conditions (artesian?) in order to make a reasonable design. Test borings should be made to confirm that elevated water pressures exist, and that they can be effectively drained. The design should determine how to discharge the collected water (French drains?), the corresponding lowered level of groundwater, and the relative FS gain. The second benefit of the vertical relief drains is that they will accelerate consolidation and shear strength gains of the slide debris. The possible use of temporary pumps in each of the vertical relief drains could provide deeper lowering of the groundwater, which may be desirable for the short term to provide quick global stability to the highway. It may be necessary to install French drains to improve drainage at the slide toe before heavy equipment is brought onto the slide mass to install the vertical relief drains.

15. There are several patterns that vertical relief drains and French drains can be installed, including herringbone, grid system, or parallel interceptor systems. Excavation for French drains could be very difficult in the soft wet clays, and flatter side-slopes should be anticipated in the trench sidewalls. Shoring systems are usually too cumbersome for this work because of the need to place a geotextile envelope around the free-draining rockfill.

16. Piezometer instrumentation and monitoring will be critical to determining the adequacy of the drainage system. The instruments should be spaced apart across the slide mass, on a grid pattern, to verify that groundwater lowering objectives have been met. A preliminary program of additional monitoring piezometers is shown on Figure 4. At present, the depth of the slide plane is still being investigated. Therefore we recommend that dual piezometers be installed to bracket the interpreted range of slide depth.

17. The French drains should be made perpendicular to the riprap bank, and should drain into the riprap as deep as possible (elevation +85 feet?). The lower the French drain the better to lower groundwater and to increase FS stability.

18. As geotechnical data on groundwater levels and slide failure plane depth are determined, and the mitigation designs are started, we should be advised to provide more specific input on the design of landslide drainage systems.

19. The long-term stabilization method for the headscarp area (to protect the highway) is currently planned to be a side slope rock buttress. However, if the landslide mass stability level (FS) is not adequate to support new fill weight at the top, then alternatives may need to be considered, such as: a) overexcavating the road subgrade and replacing with a geogrid-reinforced fill that does not put weight on the landslide, b) a soil nailed slope (1:1), or c) a tied-back or anchored wall.

5.3 Planning the Observational Approach:

1. The Observational Approach is needed because of the fast-track nature of design and construction, which does not provide the normal time for exploration, instrumentation, testing and analysis. Therefore, assumptions have been made and possibly some issues could have been overlooked. As a result, continued geotechnical investigations during design and construction need to be performed to verify assumptions and to identify where changes in the design or construction approach should be made. In addition, the landslide behavior is subject to storms and groundwater changes and other outside influences, which could result in unexpected slide movements.

2. This approach requires careful planning so that significant uncertainties can be further evaluated by geotechnical engineers, with adequate instrumentation and monitoring, and designs verified or modified as necessary.

LANDSLIDE TECHNOLOGY

By George Machan

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The geotechnical report was prepared for the use of the Owner in the design of the subject facility and should be made available to potential contractors and/or the Contractor for information on factual data only. This report should not be used for contractual purposes as a warranty of interpreted subsurface conditions such as those indicated by the interpretive boring and test pit logs, cross-sections, or discussion of subsurface conditions contained herein.

The analyses, conclusions and recommendations contained in the report are based on site conditions as they presently exist and assume that the exploratory borings, test pits, and/or probes are representative of the subsurface conditions of the site. If, during construction, subsurface conditions are found which are significantly different from those observed in the exploratory borings and test pits, or assumed to exist in the excavations, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. If there is a substantial lapse of time between the submission of this report and the start of work at the site, or if conditions have changed due to natural causes or construction operations at or adjacent to the site, this report should be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

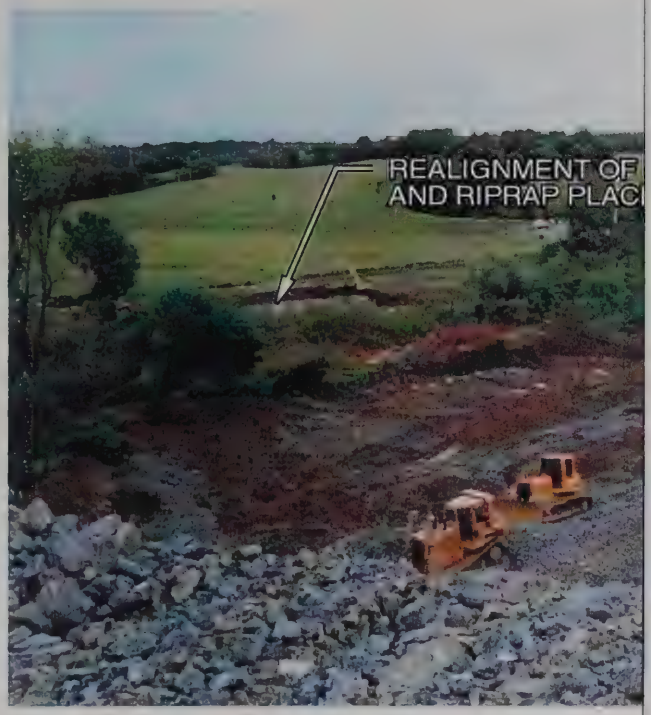
The Summary Boring Logs are our opinion of the subsurface conditions revealed by periodic sampling of the ground as the borings progressed. The soil descriptions and interfaces between strata are interpretive and actual changes may be gradual.

The boring logs and related information depict subsurface conditions only at these specific locations and at the particular time designated on the logs. Soil conditions at other locations may differ from conditions occurring at these boring locations. Also, the passage of time may result in a change in the soil conditions at these boring locations.

Groundwater levels often vary seasonally. Groundwater levels reported on the boring logs or in the body of the report are factual data only for the dates shown.

Unanticipated soil conditions are commonly encountered on construction sites and cannot be fully anticipated by merely taking soil samples, borings or test pits. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. It is recommended that the Owner consider providing a contingency fund to accommodate such potential extra costs.

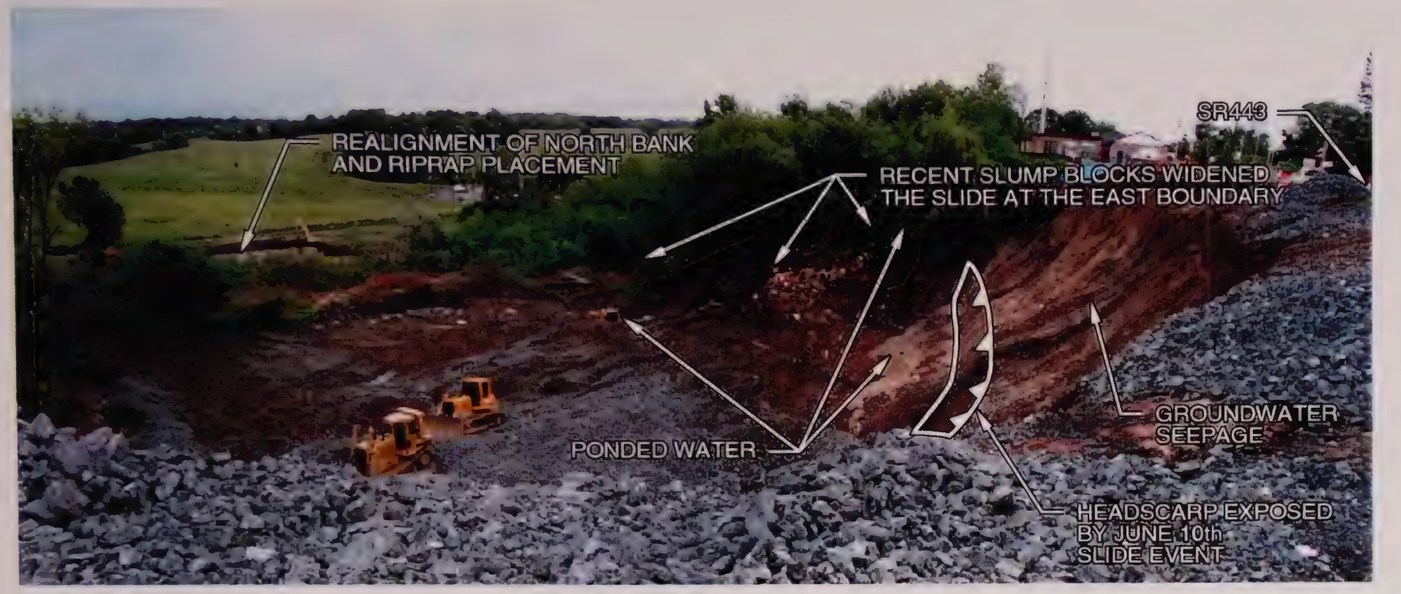
This firm cannot be responsible for any deviation from the intent of this report including, but not restricted to, any changes to the scheduled time of construction, the nature of the project or the specific construction methods or means indicated in this report; nor can our firm be responsible for any construction activity on sites other than the specific site referred to in this report.



G WEST
REAM

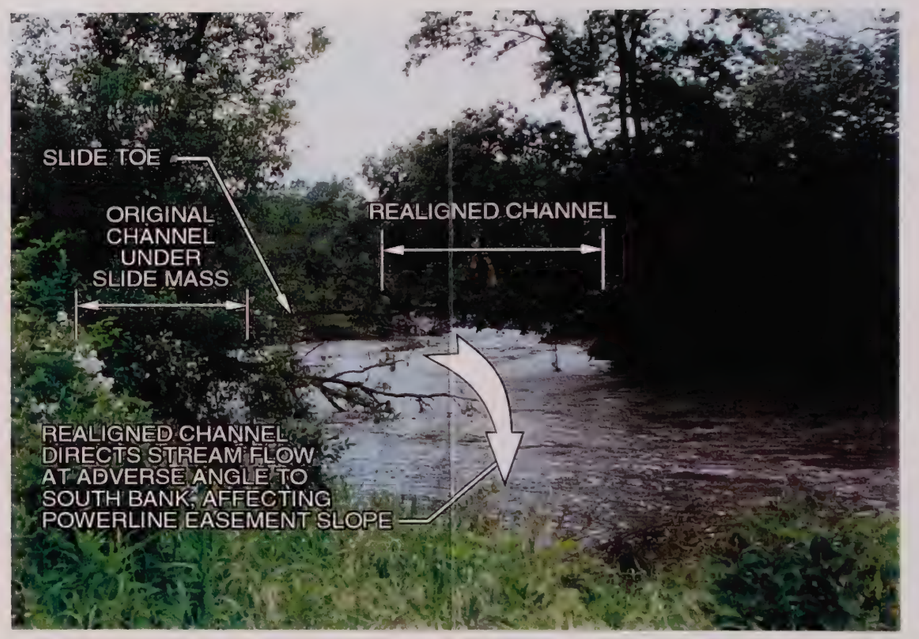
1280\UPDATE\FIGURE01 LJW

PHOTOS JUNE 15, 2000	DATE JUN 2000
	JOB NO. 1280
SR 443 DELAWARE AVE. LANDSLIDE ELSMERE, NEW YORK	FIG. 1



LOOKING NORTHEAST
AFTER JUNE 10th SLIDE EVENT

LOOKING EAST
DOWNSTREAM



LOOKING WEST
UPSTREAM

1280\UPDATE\FIGURE01 LJW

Landslide Technology 10250 S.W. Greenburg Rd. Portland, OR 97223	TITLE: PHOTOS JUNE 15, 2000 JOB: SR 443 DELAWARE AVE. LANDSLIDE ELSMERE, NEW YORK	DATE: JUN 2000 JOB NO.: 1280 FIG.: 1

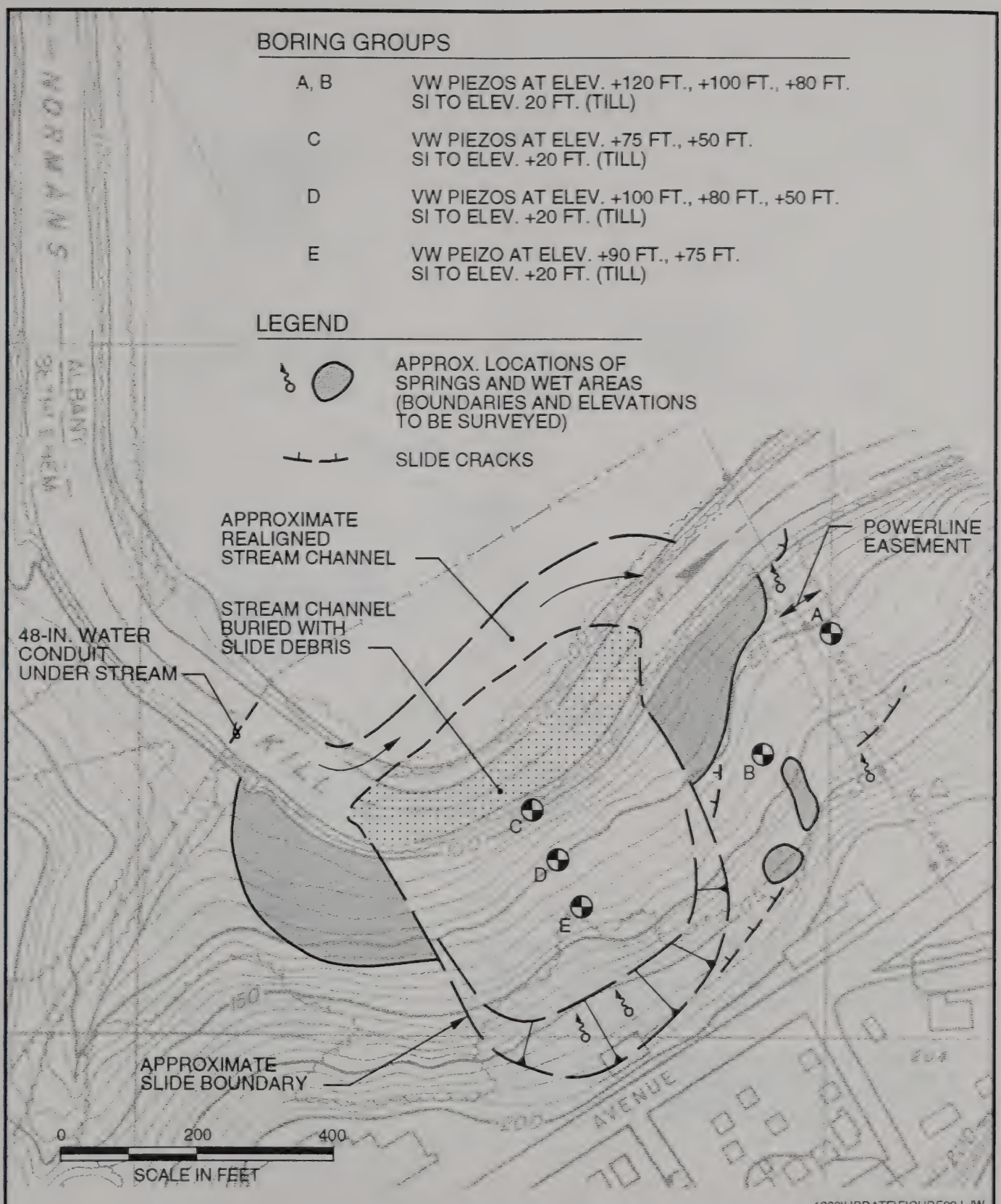
BORING GROUPS

A, B	VW PIEZOS AT ELEV. +120 FT., +100 FT., +80 FT. SI TO ELEV. 20 FT. (TILL)
C	VW PIEZOS AT ELEV. +75 FT., +50 FT. SI TO ELEV. +20 FT. (TILL)
D	VW PIEZOS AT ELEV. +100 FT., +80 FT., +50 FT. SI TO ELEV. +20 FT. (TILL)
E	VW PEIZO AT ELEV. +90 FT., +75 FT. SI TO ELEV. +20 FT. (TILL)

LEGEND

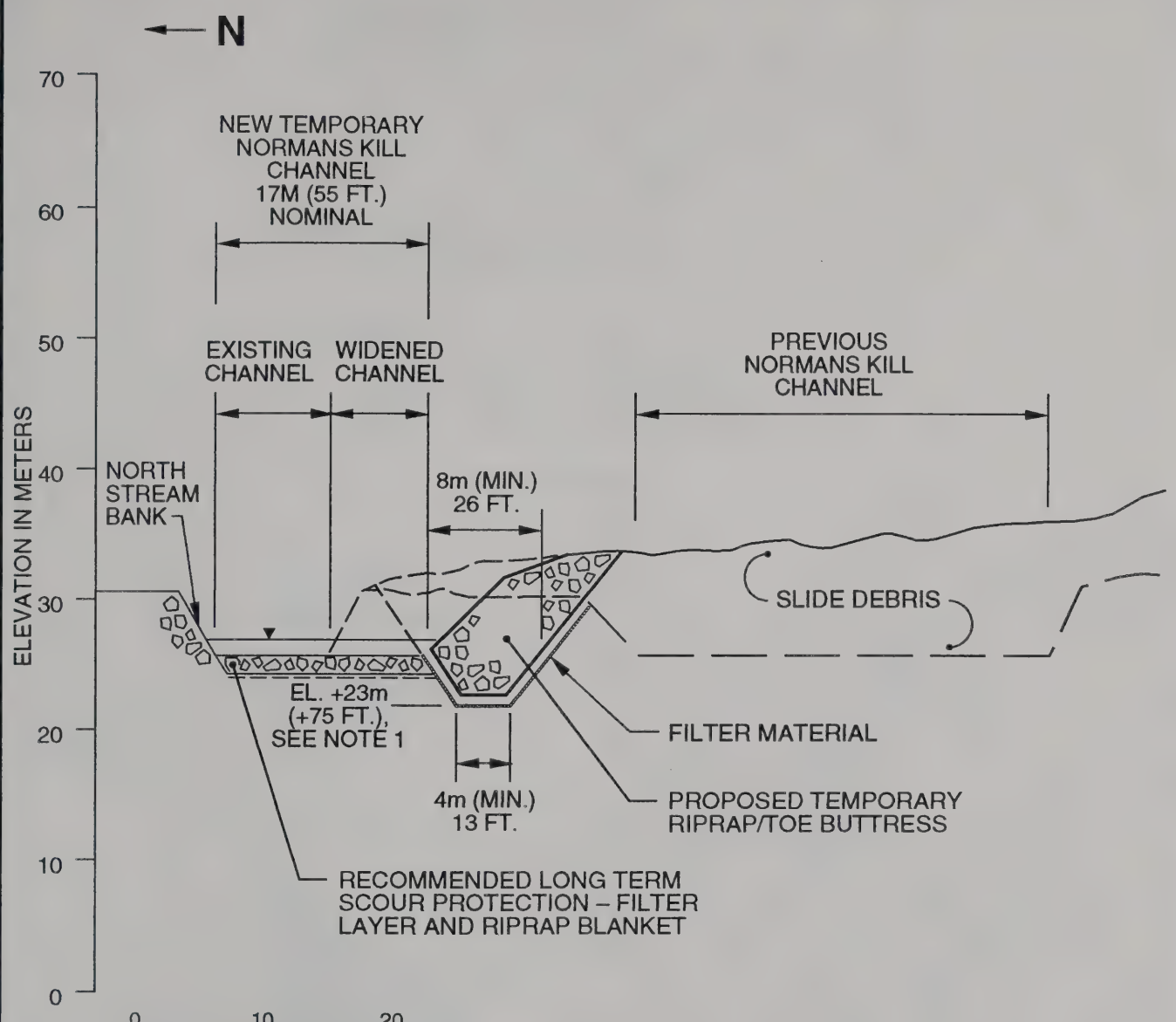
APPROX. LOCATIONS OF SPRINGS AND WET AREAS (BOUNDARIES AND ELEVATIONS TO BE SURVEYED)

SLIDE CRACKS



1280\UPDATE\FIGURE02 LJW

Landslide Technology 10250 S.W. Greenburg Rd. Portland, OR 97223	TITLE SITE PLAN: RECOMMENDED INSTRUMENTATION	DATE	JUN 2000
		JOB NO.	1280
	JOB SR 443 DELAWARE AVE. LANDSLIDE ELSMERE, NEW YORK	FIG.	2



0 10 20

SCALE IN METERS

0 40 80

SCALE IN FEET

NOTES

1. THE FOUNDATION ELEVATION FOR THE RIPRAP STREAM BANK AND THE THICKNESS OF THE RIPRAP BLANKET SHOULD BE EVALUATED BY A HYDRAULIC ENGINEER TO DETERMINE ADEQUATE DEPTH FOR PROTECTION AGAINST SCOUR.
2. THIS CONCEPT SHOULD BE REEVALUATED WHEN NEW INSTRUMENTATION DATA BECOMES AVAILABLE AND DESIGN GOALS ARE CLARIFIED. DIMENSIONS MAY NEED TO BE ADJUSTED IN DESIGN.

1280\UPDATE\FIGURE03 LJW

Landslide Technology
10250 S.W. Greenburg Rd.
Portland, OR 97223

TITLE	TEMPORARY RIPRAP CONCEPT SKETCH
JOB	SR 443 DELAWARE AVE. LANDSLIDE ELSMERE, NEW YORK
DATE	JUN 2000
JOB NO.	1280
FIG.	3

ADDED PIEZOMETERS FOR MONITORING DRAWDOWN

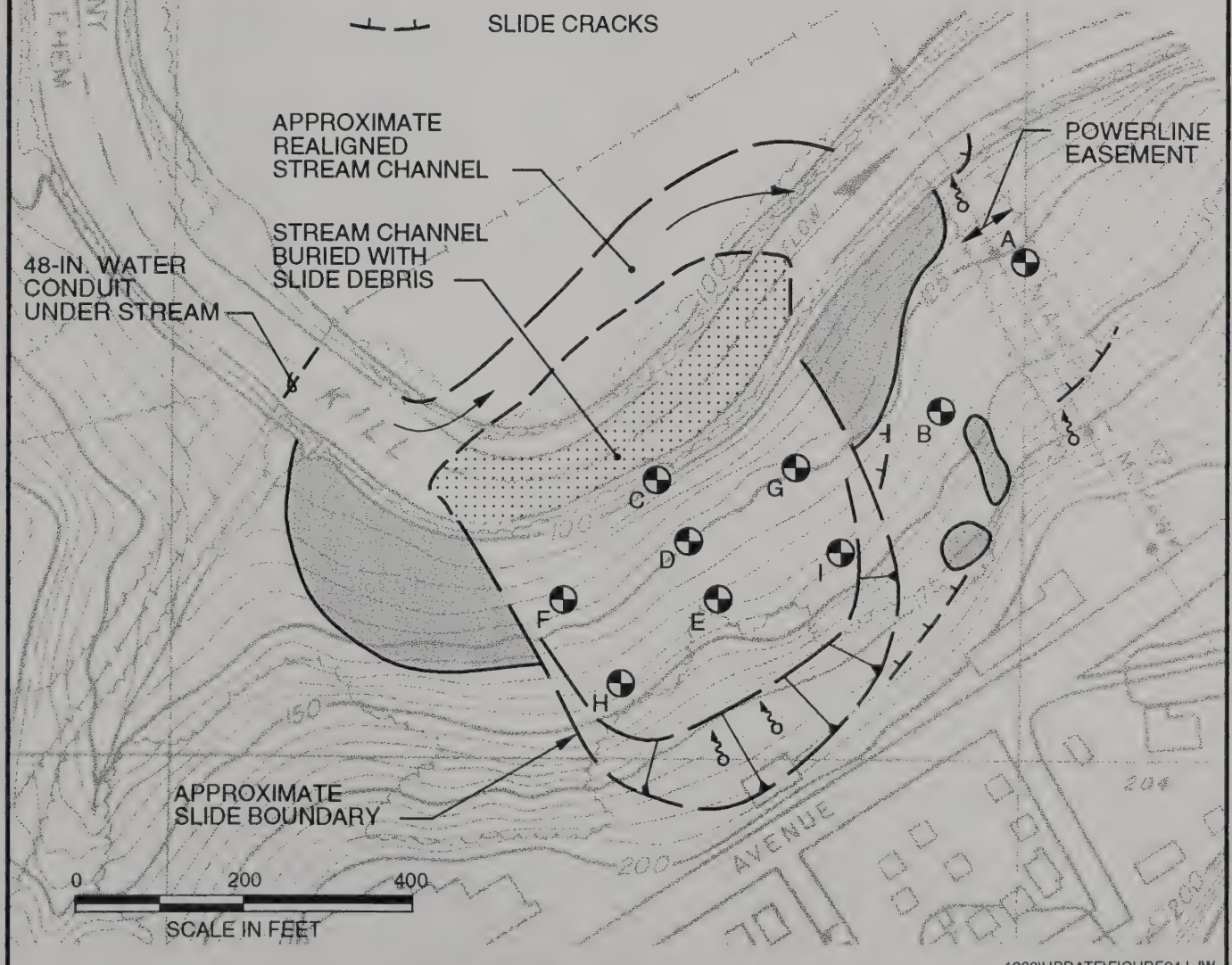
F, G	VW PIEZOS AT ELEV. +80 FT., +100 FT., (APPROX., TBD)
H, I	VW PIEZOS AT ELEV. +90 FT., +110 FT. (APPROX., TBD)

NOTE: REFER TO FIGURE 2 FOR INSTRUMENTS A – E

LEGEND

APPROX. LOCATIONS OF SPRINGS AND WET AREAS

SLIDE CRACKS



1280\UPDATE\FIGURE04 LJW

Landslide Technology 10250 S.W. Greenburg Rd. Portland, OR 97223	TITLE RECOMMENDED PIEZOMETERS TO MONITOR DRAWDOWN	DATE JUN 2000	
		JOB NO. 1280	
	JOB SR 443 DELAWARE AVE. LANDSLIDE ELSMERE, NEW YORK	FIG. 4	

APPENDIX: Updated Laboratory Test Summary

TABLE A1: SUMMARY RESULTS OF FIRST SERIES OF TESTS

Notes:
1. Borings FH-X-1, FH-X-2, and FH-X-3 are located at highway elevation.
2. Tests were performed by NYSDOT. Additional tests are being performed.

Boring, sample	Depth ft., (m)	Elev. ft., (m)	Natural Moisture Content	Liquid Limit	Plasticity Index	Effective CIU Triaxial	CU - total Stress Triaxial
FHX-3, J1	1, [0.3]	200, [61]	10%				
FHX-1, J2	5, [1.5]	194, [59.5]	33%				
FHX-2, J1	5, [1.5]	194, [59.5]	30%				
FHX-3, J2	10, [2.9]	191, [58.3]	25%				
FHX-1, J3	10, [2.9]	191, [58.3]	37%				
FHX-1, J3A	11, [3.2]	191, [58.0]	29%				
FHX-2, T2	10, [2.9]	191, [58.3]	39%				
FHX-1, J4	15, [4.5]	186, [56.7]	35%				
FHX-2, T3	15, [4.5]	186, [56.7]	40%	35%	17%	phi=18	
FHX-2, T3		186, [56.7]				c=20kPa	
FHX-1, J5	20, [6]	181, [55.2]	35%				
FHX-2, T4	20, [6]	181, [55.2]	29 - 38%	24%	6%	phi=23	
FHX-2, T4		181, [55.2]				c=20kPa	
FHX-3, J3	20, [6]	181, [55.2]	37%				
FHX-1, J6	25, [7.5]	176, [53.7]	31%				
FHX-2, T5	25, [7.5]	176, [53.7]	28%	25%	7%	phi=29	Su=207kPa @Pc=138kPa
FHX-2, T5	25, [7.5]	176, [53.7]				c=18kPa	Su=345kPa @Pc=276kPa
FHX-1, J7	30, [9]	171, [52.2]	35%				
FHX-3, J4	30, [9]	171, [52.2]	31%				
FHX-1, J8	35, [10.5]	166, [50.6]	34%				
FHX-2, T6	35, [10.5]	166, [50.6]	33%	26%	8%	phi=28	
FHX-2, T6		166, [50.6]				c=20kPa	
FHX-1, J9	40, [12]	161, [49.1]	36%				
FHX-3, J5	40, [12]	161, [49.1]	41%				
FHX-1, J10	45, [13.5]	156, [47.6]	42%				
FHX-2, T7	45, [13.5]	156, [47.6]		38%	19%	phi=19	
FHX-2, T7		156, [47.6]				c=20kPa	
FHX-1, J11	50, [15]	151, [46.1]	37%				
FHX-3, J6	50, [15]	151, [46.1]	38%				
FHX-1, J12	55, [16.6]	146, [44.5]	35%				
FHX-2, T8	55, [16.6]	146, [44.5]	38 - 45%	35%	15%	phi=18	
FHX-2, T8	55, [16.6]	146, [44.5]				c=10kPa	
FHX-2, T8	55, [16.6]	146, [44.5]					
FHX-1, J13	60, [18.1]	141, [43]	37%				
FHX-3, J7	60, [18.1]	141, [43]	35%				
FHX-1, J14	65, [19.7]	136, [41.5]	35%				
FHX-2, T9	65, [19.7]	136, [41.5]	35 -42%	34%	15%	phi=18	
FHX-2, T9	65, [19.7]	136, [41.5]				c=2kPa	
FHX-2, T9	65, [19.7]	136, [41.5]					
FHX-1, J15	70, [21.2]	131, [40]	36%				
FHX-3, J8	70, [21.2]	131, [40]	36%				
FHX-1, J16	75, [22.7]	126, [38.4]	36%				
FHX-2, T10	75, [22.7]	126, [38.4]	35 - 45%	34%	13%	phi=18	Su=209kPa @Pc=207kPa
FHX-2, T10	75, [22.7]	126, [38.4]				c=0kPa	Su=236kPa @Pc=310kPa
FHX-2, T10	75, [22.7]	126, [38.4]					Su=247kPa @Pc=414kPa
FHX-1, J17	80, [24.2]	121, [36.9]	36%				
FHX-3, J9	80, [24.2]	121, [36.9]	38%				

TABLE A1: SUMMARY RESULTS OF FIRST SERIES OF TESTS (Continued)

Notes:
1. Borings FH-X-1, FH-X-2, and FH-X-3 are located at highway elevation (approx. 200 feet).
2. Boring UDH-4 is located on the landslide mass; approx. surface elev. 136 feet.
3. Tests were performed by NYSDOT. Additional tests are being performed.

Boring, sample	Depth ft., (m)	Elev. ft., (m)	Natural Moisture Content	Liquid Limit	Plasticity Index	Effective CIU Triaxial	CU - total Stress Triaxial
FHX-1, J18	85, [25.8]	116, [35.4]	40%				
FHX-2, T11	85, [25.8]	116, [35.4]	42 - 49%	40%	18%		
FHX-2, T11	85, [25.8]	116, [35.4]	42 - 49%	42%	21%	phi=17	Su=303kPa @Pc=469kPa
FHX-2, T11	85, [25.8]	116, [35.4]				c=25kPa	Su=311kPa @Pc=552kPa
FHX-2, T11	85, [25.8]	116, [35.4]					Su=334kPa @Pc=621kPa
UDH-4, T2	20, [6]	116, [35.4]	26%	26%	7%	phi=25.5	Su=76kPa,@ Pv=124kPa
UDH-4, T2		116, [35.4]				c=20kPa	
FHX-1, J19	90, [27.3]	111, [33.9]	44%				
FHX-3, J10	90, [27.3]	111, [33.9]	33%				
UDH-4, T3	26, [7.5]	110, [33.5]	27%	35%	15%		Su=67kPa,@ Pv=138kPa
UDH-4, T3	26.4, [7.5]	110, [33.4]	27%	41%	20%	phi=23	
UDH-4, T3		110, [33.4]				c=20kPa	
UDH-4, T4	30, [9]	106, [32.3]	38%	39%	19%	phi=15	Su=90kPa@Pv=152kPa
FHX-2, T12	95, [28.9]	106, [32.3]	49%	35%	14%	c=40kPa	Su=336kPa @Pc=517kPa
FHX-2, T12	95, [28.9]	106, [32.3]					Su=236kPa @Pc=621kPa
FHX-2, T12	95, [28.9]	106, [32.3]					Su=429kPa @Pc=724kPa
UDH-4, T4	30.8, [9.4]	105, [32.1]	38%	36%	16%		
UDH-4, T4	31.3, [9.6]	105, [31.9]	38%	33%	14%		
FHX-3, J11	100, [30.4]	101, [30.8]	42%				
FHX-1, J20	100, [30.4]	101, [30.8]	38%				
UDH-4, T5	35, [10.5]	101, [30.8]	29%	34%	15%		
UDH-4, T6	40, [12]	96, [29.3]	28%	29%	10%	phi=21	Su=110kPa@Pv=179kPa
UDH-4, T6	41.2, [12.6]	94.7 [28.8]	28%	36%	16%	c=20kPa	
FHX-2, T13	105, [31.9]	96, [29.3]	39%	26%	8%	phi=17	Su=386kPa @Pc=345kPa
FHX-2, T13	105, [31.9]	96, [29.3]		29%	11%	c=50kPa	Su=428kPa @Pc=448kPa
FHX-2, T13	105, [31.9]	96, [29.3]					Su=783kPa @Pc=586kPa
FHX-1, J21	110, [33.4]	91, [27.8]	32%				
FHX-3, J12	110, [33.4]	91, [27.8]	31%				
UDH-4, T7	45, [13.5]	91, [27.7]	32%	41%	19%	phi=34.5	Su=148kPa@Pv=207kPa
UDH-4, T7	45.5, [13.9]	90, [27.6]	32%	27%	9%	c=0	
UDH-4, T7	46.2, [14.1]	90, [27.4]	32%	24%	5%		
UDH-4, T8	50, [15]	86, [26.2]	30%				
UDH-4, T9	55, [16.6]	81, [24.7]	28%				
FHX-1, J22	120, [36.5]	81, [24.7]	30%				
FHX-3, J13	120, [36.5]	81, [24.7]	33%				
UDH-4, T10	60, [18.1]	76, [23.2]	38%				
UDH-4, T11	65, [19.7]	71, [21.6]	31%				
FHX-1, J23	130, [39.5]	71, [21.7]	28%				
FHX-3, J14	130, [39.5]	71, [21.7]	30%				
UDH-4, T12	70, [21.2]	66, [20.1]	26%				
UDH-4, T13	75, [22.7]	61, [18.6]	32%				
FHX-1, J24	140, [42.5]	61, [18.6]	27%				
FHX-3, J15	140, [42.5]	61, [18.6]	39%				
UDH-4, T14	80, [24.2]	56, [17.1]	24%				
UDH-4, T15	85, [25.8]	51, [15.5]	27%				
UDH-4, T16	90, [27.3]	46, [14]	24%				
UDH-4, T17	95, [28.9]	41, [12.5]	49%				
UDH-4, T18	100, [30.4]	36, [11]	41%				
FHX-1, J25	170, [51.7]	31, [9.5]	9%				
UDH-4, T19	105, [31.9]	31, [9.4]	37%				
UDH-4, T20	110, [33.4]	26, [7.9]	31%				

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